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BLAST RESISTANT BUILDING FRAMES

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BLAST RESISTANT BUILDING FRAMES

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SYNOPSIS

The primary purpose of this paper is to make specific suggestions for good practice that will reduce damage suffered by industrial building frames if they are subjected to enemy bombing attacks. The suggestions are intended to apply primarily to a particular type of industrial building meeting the following restrictions.

1) One story Steel frame construction.
2) Building contains heavy machinery or other durable contents not liable to serious damage as a result of high winds of tornado proportions.

These criteria define a class of construction in which considerable improvement in resistance to atomic bomb attack can be made at relatively small increase in cost.

The nature of the structural effects of the atomic explosion are discussed with emphasis on the wind gust aspect of the blast wave.

The paper includes a discussion of general factors favoring maximum plastic resistance of steel beams, columns and details, applicable to all classes of steel construction wherein increased resistance to bomb attack is desired. Observance of such rules of design will also result in improved resistance to earthquake and other kinds of shock load wherein plastic strength is important. A design example is presented.

INTRODUCTION

The ebbing backwash of water from the beach revealed a small wooden box half buried by the sand, but a moment later it was engulfed by the next comber breaking over it. At this moment, in several important respects, but on a miniature scale, the structural experience of the box was similar to that of a building in the path of an atomic blast wave. The initial shock of impact of water against the box tended to break in the front side and force it along with the wave. But, rooted in the sand, it held and a moment later was engulfed on all sides by the water—a much denser medium than the air to which it had previously been exposed. The pressure of the water tended to crush in all sides of the box. In addition to the pressure, after the front of the wave had passed, the water current itself tended to drag the box along and uproot it from its buried position. If, in place of the wooden box, a small wire cage were substituted, the water would rush right through the openings and while

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the drag force on the individual wires would tend to carry the cage along, there would be no problem of design against the pressure in which the cage was engulfed.

The foregoing is well summarized in the publication "Effects of Atomic Weapons"[1] which, on page 116, states: "It is seen that two factors operate to destroy the structure, the crushing effect of the pressures developed on the structure as it becomes surrounded by the shock wave, and the aerodynamic drag of the air mass or wind motion behind the shock front." Drag force (neglecting skin friction) is, in fact, the integrated total force component of the surface pressure components resolved into the direction of the undisturbed flow. It is, roughly speaking, the translational force on either an individual member, or, in a closed structure, on the entire frame, resulting from differential loading on the front and rear faces.

The foregoing illustrations exemplify two alternate approaches to bomb resistant construction. The first would completely enclose a structure so as to protect the interior contents. Such construction is certainly desirable when human life is involved, such as in the construction of bomb shelters, or in key building units such as hospitals, telephone exchanges, power plants, etc. However, in some special cases, for example, industrial buildings housing sturdy and heavy equipment, such as presses and forges, there would be little point in attempting to protect all of the interior contents. Indeed, such protection would be economically out of the question. Small and relatively inexpensive separate bomb shelters could adequately protect workers and key items of equipment.

Since it must be expected that industrial buildings will be a prime target in the event of any atomic warfare it seems urgently desirable that improved resistance be built into such structures as such practice will greatly reduce the area of complete destruction regardless of bomb size. Furthermore, much improvement can be attained with relatively small additional cost.

Although some of the suggestions made herein have been made before, as for example, suggestions of the Federal Civil Defense Administration, the ideas presented herein have in general been corroborated independently during the past four years by the University of Michigan's Civil Engineering Department, which has conducted a number of small investigations pertaining to industrial building construction in relation to its behavior under atomic blast.[2] Emphasis has been on the drag type problem wherein the blast wave (as in the case of the wire cage on the beach) is permitted to enter and pass through the building. There is a considerable amount of information already available through the Federal Civil Defense Administration concerning the design of structures that must be fully enclosed, but relatively less information has been provided regarding the much more economical possibility of improving the design of ordinary industrial buildings whose walls are "frangible." However, the use of frangible walls where suitable has been recommended by the Federal Civil Defense Administration.

Maximum Structural Strength and Deformation

Initial general remarks on the fundamental nature of plastic strength will serve as a preliminary to the presentation of specific design suggestions.

Strength of Steel under Shock Load

Basic to the development of maximum shock load resistance is the full utilization of the plastic strength of steel.

In a tensile test, structural steel passes its upper yield-point at or above the specified minimum yield stress of 33,000 psi and a corresponding strain of approximately 0.0011. The stress then commonly drops to the "lower yield point" and strain then increases considerably with little or no further increase in stress. The strain at lower yield point of steel is usually 10 to 20 times that of the elastic strain, or, between 0.01 and 0.02. Thus, the energy that steel can absorb, per unit volume, in the initial lower yield point range, is 20 to 40 times the elastic energy per unit volume stored up to the yield point. In evaluating the plastic strength of a steel frame it is this lower yield level that is all-important. The lower yield level is principally affected by (1) time rate of strain and (2) temperature. Increased rate of strain will increase the lower yield level and, thereby, increase the plastic load-resistance of a steel structure. Studies have been made regarding the rate of strain to be expected in a steel structure when it is subjected to a sudden shock load such as might occur during an atomic blast. The rates of strain are rather high for light exterior panels but are much lower for heavier main members to which the load is more gradually transmitted as it passes from panel to girt to main member. Strictly speaking, one might be justified in using different yield points for the same steel depending on just what part of the structure were being analyzed. However, the uncertainties of the whole problem indicate the desirability of using a single yield-point in most dynamic structural analyses. Available studies from a large number of mill tests of structural steel indicate an average yield-point of about 40,000 psi. This yield strength level must be discounted somewhat because of several facts.

1) Quite reasonably, the ASTM Standard Specifications permit a rather rapid strain rate to be used in mill tests and it has been estimated that this causes an increase of roughly ten percent to the basic lower yield strength that would be obtained at a very slow rate of strain.

2) Acceptance tests for structural steel rolled beams are usually made from web material whereas the flange material is of prime importance. The web material sometimes has a slightly higher yield-point than the flange material because it is thinner and has a more rapid cooling rate.

3) In some cases the mill rest reports the upper yield-point rather than the lower yield-point and the latter is of most importance.

However, when these adverse factors are considered in conjunction with the possible rates of strain to be expected under suddenly applied shock loads from atomic blasts, a small net increase in average yield strength to be used in estimating the plastic behavior of structures seems warranted and the authors recommend 42,000 psi as a proper yield stress level to be used in making a plastic design on the basis of dynamic analyses.

Strength of Steel Beams

Tests show that the simple plastic theory predicts very well the bending strength of ordinary steel beams, especially if they meet certain minimum proportions regarding the flange and web width-thickness ratios. The elastic bending-moment capacity of a steel beam is the section modulus multiplied by the yield stress. The real, or plastic moment capacity is between 10 and 20 percent greater than the elastic capacity thus predicted by the section modulus. Steel beams with stocky proportions, that is small width-thickness ratios, may be expected to maintain their full plastic moment capacity when

bent into curvatures far greater than that at initial yield. This is the quality of the steel beam that makes it possible to realize an increase in strength in continuous steel frames over that predicted by initial yield. Furthermore, it is possible to apply simple procedures for estimating this maximum frame strength.(3)

Slender steel beams, when laterally unsupported, may buckle laterally with a combination of twisting and lateral bending. This type of behavior should be avoided, insofar as possible, when attempting to develop maximum plastic resistance. The way to avoid lateral buckling of steel beams is to provide frequent lateral and torsional support especially at points of maximum bending moment. In the American Institute of Steel Construction specification the quantity Ld/bt is given as the basic parameter of lateral-torsional buckling. In elastic design, if Ld/bt is less than 600, the full allowable working stress of 20,000 psi is permitted in designing steel beams. Ideally, a laterally unsupported steel beam with an Ld/bt ratio of 600 would buckle plastically as soon as the yield-point is reached. Actually, tests show that it may buckle at an even lower stress than the yield-point stress but this is due to the presence of residual stresses and other accidental contributing factors that tend to decrease the strength.[3] However, if the Ld/bt ratio is made considerably less than 600, an unsupported steel beam will tend to retain its strength into the plastic range to a greater and greater degree as the Ld/bt ratio is decreased. Thus, maximum plastic resistance in bending can be obtained by keeping Ld/bt between points of lateral and torsional support just as small as is possible consistent with the practical features of the design under consideration.

In the case of slender steel beams supporting wall or roof panels, respectively denoted as girts and purlins, an additional reserve of plastic strength over and above that resulting from bending alone may be worth considering. If the ends of such a member are held against longitudinal movement, large elastic and plastic deflections of the beam will result in the development of a resultant tensile stress in the total cross-section. In other words, the beam, after largely using up its bending resistance will, if held at the ends, act as a cable or rope and this resistance may in fact ultimately exceed that provided by the bending strength alone. To realize this additional strength it is essential that the ends of the beam be attached by connections that will develop the full plastic yield strength of the member in pure tension. In addition the frames must be held laterally against the forces that would be imposed by girt tension. A concrete roof slab, for example, would hold individual frames in position. If the overall tensile load-deflection curve of the member, including the deformation of the connections, is known, it is possible to predict the additional lateral load that can be carried by the member. A simple design rule will be presented.

Strength of Steel Columns

A long steel column buckles "elastically." Its strength is determined for any given cross-sectional area and length simply by the bending stiffness "EI." The strength of a very short column, however, is determined very largely by its yield strength. In the intermediate range between short and

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3. In actual practice, as a result of connection angles, end-restraint conditions usually prevail that tend to counteract this decrease in strength.

long columns, collapse occurs shortly after the yield strength of the material is reached. On the other hand, as a column gets very short it will maintain its yield strength for a considerable amount of strain, the amount depending upon the relative shortness of the column. Rules for predicting the maximum slenderness ratio of a column that should be permitted for a desirable amount of plastic yield strain will become available in the future. However, the "columns" in a one storey industrial building frame are relatively more stressed in bending than by direct stress, hence should be thought of as "beam-columns."

Beam-Columns

A beam-column is a member that simultaneously acts both as a beam and as a column. The vertical column in an industrial building frame may be considered as a beam-column since it not only carries vertical load of the roof and crane but, in addition, must carry bending moments introduced by a crane girder support and lateral wind loads on the building. As in the case of independent beams or columns, lateral support of beam-columns should be as frequent as practicable. Such support is particularly important at locations of large bending moment.

In the design of vertical members of an industrial frame building, fixed column bases are recommended, with the foundations designed to develop the full shear strength and plastic bending resistance of the steel columns in question. This practice will not only increase the plastic resistance of the column itself but will also markedly increase the lateral plastic resistance of the whole frame. In addition, if the top of the column can be restrained against rotation about both axes by members that frame in both directions, the plastic strength capacity of the column will be increased by reducing its tendency toward lateral buckling.

Compression Details of Beams and Columns

It is well known that when a plate element carrying compression or shear load has a large width-thickness ratio the plate element will buckle. For very large width-thickness ratios (as in the case of the very slender column) the buckling will be elastic. Design specifications for heavy steel construction quite generally require width-thickness ratios small enough to insure (with some factor of uncertainty included) that the yield-point will be reached before such elastic buckling takes place. Such a specification is comparable to requiring the slenderness ratio of a column to be less than, say 60 or thereabouts, or the Ld/bt ratio of a beam to be less than 400, or thereabouts. As in the case of a beam or a column, compression plate elements designed with width-thickness ratios that just meet conventional specification requirements will buckle shortly after the yield-point is reached, with an immediate falling off of the load. A beam with flange and web proportions just meeting these limiting specification ratios would buckle locally when the maximum stress due to bending reached the yield-point and thus, due to local plastic buckling, such a beam would not maintain a bending moment equal to or above that corresponding to yield-stress as bending of the beam progressed. In the future there will become available rules of practice[4] for width-thickness ratios that will insure that inception of plastic buckling will be delayed until

4. For a more detailed statement refer to a progress report on research at Lehigh University.(4)

plastic strains are developed in a considerable magnitude and thus the bending or compressive strength of a beam or column can be maintained into the plastic range and the full potential plastic strength of the structure realized. Roughly speaking, the width-thickness proportions of plates will be about half of those now permitted in conventional elastic design. There will be a considerable number of existing rolled shapes that will meet such a specification and the recommendation herein is to use members that have as small width-thickness ratios as are available in sections having the required section moduli and other properties for the designs under consideration.

Steel Connections

For the maximum resistance to shock loads, connections between members should be designed to develop the full plastic strength of the weakest connecting member in bending. In other words, full continuity of construction is desirable for maximum plastic resistance. In corner connections it is particularly desirable to introduce diagonal stiffeners so that undue shear yield causing large local angle change between connecting members will be prevented. Similarly, connections for tension members should develop the yield strength in the main body of the member.

Blast Loads

This paper will merely summarize in a general way those features of blast loads that are important from a structural point of view.

Pressure and Drag

During the small fraction of a second immediately after the first contact of the shock front against the building structure, portions of the structure farthest from the origin of blast may still be under normal conditions of pressure, air density, and air motion. The portion of the structure nearest the origin of blast is enveloped in a medium of air of greatly increased density and impinging on all sides of the structure are rapidly changing but very high pressures. Furthermore, the air is in rapid momentary motion away from the origin of blast, causing in effect, a high wind with velocities comparable to those experienced in tornadoes. If the structure is completely enclosed by heavy reinforced concrete walls and well anchored to the earth, the air motion may in some cases be a minor effect compared to the air pressure that developes momentarily around the whole structure. On the other hand, if the structure is not well anchored or is tall, such as a tier building, the net translational force, or overall drag on the entire structure, may be of paramount importance. Rules for predicting the average pressures on the front, top, and rear areas of a structure under such conditions have been presented by the Federal Civil Defense Administration.(5)

If the walls can be almost instantaneously destroyed, such as would be true for corrugated asbestos siding, the shock front will pass through as well as over the structure. If the blast disturbance passes through the structure the various structural members such as beams, columns, etc. are enveloped with the same high pressure that would destroy the walls and roof of an ordinary building. However, on the solid structural member such pressures have no damaging effects since even in the case of wood they are far below the strength of the material. However, as a result of the air flow or "wind" there is a resultant lateral force applied to the structural member in the

direction of the air flow. The average resultant force per unit area (termed "drag pressure") is at least similar to that experienced in ordinary wind storms. In a steady wind, the average force per unit area on the member depends somewhat on the shape of the member itself and on other factors such as the Reynolds Number. The drag force per unit area is equal to the product of the coefficient of drag "C_D" and the "dynamic pressure" which is equal to the kinetic energy per unit volume of the air in motion, $\frac{1}{2}\rho v^2$. Although the blast effects must be similar to those in ordinary wind there are certain differences that should be recognized. Since the blast wind is, in effect, a momentary gust, over at most in a matter of seconds or, in some cases, a fraction of a second, there may be some question as to whether or not drag coefficients that have been obtained in wind tunnels under steady flow conditions may be applied with any degree of accuracy. In addition, it must be remembered that the density of the air is much greater than ordinary atmosphere and that the average pressure on the object under drag is also greater than ordinary atmospheric pressure on all sides.

Dynamic Structural Analysis

In general, the design of a structure subjected to shock loads should be checked by a dynamic analysis. The dynamic analysis is one in which the essential features of the structure, involving its mass and load-deflection characteristics, are reduced to the simple equivalent system to which the expected force, variable as a function of time, is applied. Then, by means of the law of motion ($F=Ma$) the deflection of the structure including permanent set, if any, is determined as a function of time.

If a dynamic analysis is at all realistic, the magnitude, duration, and other pertinent characteristics of the shock load must be known as a function of time. The load-deflection history of the structure must also be predictable. In the case of a structure which may be subjected to nuclear bomb blast, the great uncertainty as to the size and location of the actual bomb burst, together with other uncertainties involving the orientation of the building plan with respect to the direction of the blast wave, the effects produced by adjacent structures, the actual drag coefficients, the rate of build-up of drag, and rate of dissipation of blast effect, all add up to a situation in which precise calculations are not warranted except in precisely controlled atomic test operations.

The goal with respect to industrial building frame construction should be to provide the maximum possible increase in lateral resistance to deflection for the minimum added cost. Increased resistance must be accompanied by careful attention to all critical design details so that there are no weak links in the chain of dynamic force transfer and accompanying elasto-plastic resistance. The result of such design will be the reduction in area of total destruction for any given bomb size and location. Considering the nominal atomic bomb used as a basis for the book "Effects of Atomic Weapons"(1) and similar to that dropped over Hiroshima and Nagasaki, severe damage to industrial frame buildings which had their walls stripped away resulted for light frame construction at distances up to 5000 feet from ground zero. Studies of typical designs of similar steel structures according to current practice in this country show expected severe damage at about the same distance from ground zero. However, studies have shown that with very little increase in cost this zone of severe damage to the skeleton framework can be reduced to less than

3000 feet from ground zero. This means a reduction in area of total damage of almost four times. Thus, for a relatively small increase in cost of construction the area of total destruction under a given bomb might be reduced by a factor of 4. Such an approach is certainly worth serious consideration.

It is true that the hydrogen bomb greatly increases the diameter of the damage area. Nevertheless, comparable reductions in the area of total destruction would result by use of structures designed with improved plastic strength. The possibility of improving chances of survival is no less to be desired in the case of larger yield weapons.

It was stated that, in view of the uncertainties involved, precise dynamic analyses of the behavior of structures seem unwarranted insofar as application to improvement in design is concerned. This statement can conveniently be examined in greater detail by reference to the paper by N. M. Newmark,(6) in which in a single chart the maximum elastic or plastic deflection can be approximately determined for a simple "elasto-plastic" resistant structure (See Fig. 1) subjected to an initial peak loading with linear decay that is somewhat similar to that experienced by the structure under bomb blast. (See Fig. 2). In Fig. 6 of reference 4 the ratio of maximum deflection to deflection at the yield limit of the steel is given in terms of two parameters. One of these (P_m/Q_m) is the ratio of peak applied load to maximum plastic resistance of the structure. The other (t_1/T) is the ratio of duration of the simplified equivalent blast wave to the natural period of the structure. Let us assume as a tentative basis for a simplified approach that the structure will be designed for a lateral strength exactly equal to the estimated peak lateral dynamic load (P_m) for which survival of the skeleton structural frame is desired. The duration of the blast wave may vary from about six-tenths of a second up to ten times as much for a bomb one thousand times as powerful.(5) However, the shape of the drag-time curve is not at all triangular but is approximately as shown in Fig. 2. Thus the effective duration for the equivalent load would be roughly one-half the total duration. This value is selected for comparative purposes but the triangular load-time curve thus chosen has approximately the same total impulsive effect as the actual curve. Assume then that the effective duration of the equivalent load is between 0.3 seconds for a nominal bomb and 3.0 seconds for a bomb one thousand times more powerful. Now for a one-story industrial steel frame structure, the natural period of vibration probably will range between extreme limits of 0.2 and 1.0 seconds. Thus, for the entire specified range of bomb sizes and periods of structure the ratio of equivalent duration to period of structure (t_1/T) might range from 0.3 to 15. Although this is a large range, Fig. 6 of the Newmark(6) paper shows that the deflection ratio, that is, the ratio of maximum deflection to yield deflection, ($\frac{x_m}{x_e}$, Fig. 1) for such conditions would range between 0.8 and 10. Thus the extent of damage would lie between none at all (with a safety factor of 1.25) and a deflection ten times the yield deflection. The latter would represent rather severe permanent deflection of the structure but would be definitely short of collapse since a plastically designed continuous frame steel structure can easily withstand deflections ten times as great as the elastic deflection (x_e) and can be pulled back into shape by competent rigging procedures. Interestingly, for the particular case of $Q = P_{max}$, there is very nearly a linear relationship between the logarithms of $\frac{x_m}{x_e}$ and t_1/T, expressible (using base 10 logarithms) as follows:

$$\log \left(\frac{x_m}{x_e}\right) = 0.2558 + 0.5924 \log (t_1/T)$$

Structural Design

On the basis of the foregoing, the following is suggested as a tentative basis for design of the particular class of structure herein under consideration.

> Industrial steel frame buildings of one-story height housing heavy equipment that can with little damage be exposed to heavy blast waves shall be designed so that all exterior and interior wall coverings are of frangible material, easily destroyed under pressures somewhat greater than those expected from maximum wind storm. In other words, the exterior and interior wall panels should fail at pressures of not more than 100 to 150 pounds per square foot. This would provide a very adequate margin of safety for conventional design loads of 20 to 30 pounds per square foot. The main frame and bracing, if any, should be designed for a plastic ultimate strength that will resist a load of 1400 pounds per square foot of skeleton area (projected in elevation) of all parts attached rigidly to the main frame.

The value of 1400 pounds per square foot has been selected as corresponding approximately to the expected maximum drag pressure at 3000 feet from a nominal atomic bomb[1] and using a drag coefficient of slightly less than two. It is recommended that, in view of all the uncertainties that are involved, no allowance be made for drag shielding. By this is meant that if successive purlins are placed one behind another under a flat roof, the sum total projected area of all the individual purlins involved will be included. The equivalent load at the roof level equals the total load applied at the roof level plus one-half of the total load estimated as applied on all the wall skeletons. A simple illustrative design example is included in an appendix. The level of 1400 pounds per square foot is not as drastic as might be imagined since the skeleton area is usually much less than the solid wall area considered in normal design and the proposed design is for ultimate strength rather than for working stresses.

A number of more general studies and recommendations have been made regarding proper procedures for reducing damage as a result of atomic blast.[5,7,8,9,10]

The following further recommendations, as previously stated, will be applicable primarily to one-story industrial buildings wherein the use of frangible walls seems desirable. These recommendations naturally will include many ideas that have been suggested previously but will summarize the concepts that have been checked by studies made in various projects on this type of construction at the University of Michigan during the past few years.

It is well to again emphasize the obvious fact, that when frangible wall construction is used, adequate personnel shelters should be provided adjacent to or within the structure. Recommendations for the construction of such shelters are available through the Federal Civil Defense Administration and elsewhere.[9]

Wall Construction

Wall construction should be "frangible." That is, it should fail—preferably disintegrate-under pressure differences from either direction that are somewhat in excess of the normal wind design loads multiplied by a suitable factor of safety. Corrugated asbestos siding, plastic, or some other nonmetallic wall siding material would seem desirable but the stipulation "frangible" does

not necessarily rule out corrugated steel or other of the many types of steel or aluminum wall sidings that have been used. If steel siding is used the possibility is suggested that it might be hinged at the top and lightly attached at the bottom so that under loads in excess of wind loads panels would rotate inward or outward and permit immediate access of the blast without undue wall damage. Hinged glass windows also could be used, with preference given to the heavy industrial type glass with wire mesh embedded therein. If steel siding is used with ordinary clip or bolt attachment the time for failure at distances for which structural survival is anticipated will be less than 0.005 seconds.(2) Thus, even though a very considerable load is momentarily transmitted to the frame, the overall dynamic impulse to the frame will be but a small fraction of the total impulse received by virtue of the drag load on the skeleton frame during the much greater duration of the blast. A number of analyses have been made in which the impulse of the siding was included. In the case of the type of construction under consideration, even for relatively weak main frames, the error in calculated maximum permanent deflection was only a few percent when the effect of siding impulse was neglected. Obviously, however, when metal siding is ripped from its place of attachment by high pressure, then carried along by wind velocities of several hundred miles per hour, it becomes in itself a missile of considerable hazard to equipment and to parts of the main building structure. Thus, the goal should be either for automatically venting windows and siding or completely frangible siding.

Siding is usually attached to horizontal girts and these in turn are attached to the main building columns. It is sufficient for usual design loads to attach girts by two rivets or bolts at each end. However, it seems desirable to improve as much as possible the load resistance of the girts since, if they are in a region wherein they survive with little damage, they can be reused for new siding during reconstruction after bombing. Saving in time would be more important than saving in cost. Studies made by the authors show that the usual attachment at the ends of the girts will not develop maximum girt resistance induced by indirect tensile forces as the girt deflections get large. Initially the girt acts as a beam but it gradually changes into action as a cable if the ends are strong enough to develop such cable action. Some improvement can be made by adding to the number of rivets or bolts in the end attachments but the best type of construction for girts is to make them completely continuous beams with butt welds at all splices along their length. Thus, for a given size girt not only will the resistance due to bending be greatly increased because of fixed-beam action and plastic redistribution of moments but, after bending resistance is exhausted, the girts will mutually hold each other by the ends and develop into a multiple cable system. Studies made by the authors show that such construction can multiply the lateral-load resistance of girts by a factor of as much as ten with very little increase in cost of construction.

As an approximate procedure for the design of girts, taking advantage of membrane or "cable" resistance, it may be assumed that the deflection curve of a girt is the arc of a circle. Let it also be assumed that a permanent center deflection of 1/30th the total span is permissible as a basis for plastic strength design. If "Q" represents the total load on one girt, for this particular deflection ratio, the tension in the girt, assuming no help at all by bending resistance, would be 3.75Q. Assume, arbitrarily, that $T = 4Q$ be a basis for design.

The drag load on the girts may be considerably less than the pressure

load at a greater distance from ground zero where the panels do not fail. To illustrate, assume that the girts are 7 inch 9.8 pound channel sections (as was used in the illustrative design[5]) in which case the "design" drag load in pounds per foot, using the recommendation herein suggested, would be equal to 244 pounds per linear foot. If the girts are placed at 5'6" centers, supporting corrugated asbestos paneling, that will fail at, say, 150 pounds per square foot (although a lesser failure load might be desirable) the load per linear foot on the girt would be 825 pounds. Thus, a girt may be much more severely loaded by pressure at a great distance from a bomb burst than it would be by drag at a much nearer distance. To avoid undue damage at great distances, girts should be designed either for the total load transmitted by the panels at failure or, for a drag load on the bare girt corresponding to 1400 psf, whichever is the greater.

Roof Construction

One of the beneficial effects of using frangible wall construction is the fact that the blast wave may enter the building with but a momentary delay and thus the pressure on the top side and under side of the roof will be equalized. This minimizes tendency for roof failure and it will be especially favorable if the roof is flat or nearly so.

Experience during the last war(7) brought out the fact that less overall damage resulted and rehabilitation of factories could proceed most rapidly if the roof remained intact after a bomb raid. This was particularly the case if heavy rains were experienced shortly after the attack. Since the use of frangible walls makes feasible the design of flat roof construction for usual design loads with a good chance of roof survival under atomic bombing this advantage should be kept in mind. Although not essential to the proposed concept of design, the use of a solid reinforced concrete roof slab four or five inches thick seems desirable and should have the following advantages.

1) The additional mass increases the inertia resistance, reducing the acceleration under impulse.
2) In continuous frame construction the added weight of the roof will require heavier column members which will, in turn, improve the lateral strength resistance even if the bending moments for normal vertical design loads are the critical ones. Of course, if the building is in a region governed by earthquake design codes, the addition of mass to the roof structure will require considerably heavier columns and, therefore, may not seem to be warranted.
3) Even a relatively thin reinforced concrete roof slab will provide a certain amount of protection against radiation hazard. This will be desirable if bomb shelters are located within the building structure and, in addition, will minimize fire hazard after bombing.
4) As mentioned previously, the reinforced concrete roof will provide protection from the weather for building equipment during the period of rehabilitation after exposure to bombing.
5) A solid reinforced concrete roof will provide lateral strength and stiffness to the purlins or beams that support the roof and will also tend to hold the entire building structure together as a unit.
6) If cross bracing can be provided in the end panels of the building the roof may act as a horizontal beam transferring the drag loads to the

5. See Appendix.

stronger ends of the structure and thus minimizing the amount of load carried by the building bents themselves. The horizontal beam action will also relieve the column base connections and footings in the interior portion of the building from excessive lateral and over-turning forces that would otherwise exist.

7) The recommendation to use reinforced concrete roof slabs applies primarily to relatively narrow buildings. In a very broad building it probably would be desirable to intersperse concrete roof areas with frangible roof areas to insure overall pressure equalization. In a building with strong interior partitions or with massive industrial equipment blocking free passage of air through the building, frangible roof construction would be preferable to reinforced concrete.

In general, a flat or nearly flat roof is to be preferred over a steeply sloping roof for a number of reasons.

1) Less vertical projected area is exposed, thus greatly reducing the total drag load on the roof for flat construction as compared with peaked construction.
2) If a peaked roof is used there will not be the same degree of pressure equalization on the upper and lower sides as the blast wave passes through the building. Furthermore, although little information seems to be available on this subject, blast wave reflections will develop from the lee underside of the peaked roof structure, modifying the pressure-time relationship and tending more toward roof failure than would be true for the flat roof.

Main Frame

In a previous paper(2) a considerable number of different designs of single bay industrial building bents were compared as to lateral resistant strength and behavior at different distances from a "nominal" atomic bomb burst. A 40 pound per square foot roof live load was used as the basis for all of the designs. No attempt in these studies was made to design for a lateral blast load. The comparisons included a continuous welded frame and a truss frame and each was designed alternately for either light-weight or reinforced concrete roof slab. For each of these four cases two different column base design assumptions were made, (1) pinned and (2) fixed. As a further variable, all eight basic designs were analyzed with and without exterior reinforcing strands anchored in the ground and attached near the top of each bent.

The following will summarize the relative lateral load resistances of the sixteen different designs that were considered. For the eight cases without any special strand reinforcement the continuous welded steel frame with heavy roof and fix column bases had a maximum plastic resistance approximately 17 times that of the weakest design, namely, the truss frame with lightweight roofing and pinned column bases. Obviously, the great difference in lateral resistance for these frames of conventional design is completely out of proportion to cost differentials that are involved. As mentioned previously, all were designed for the same vertical loads. In brief, fixing the column bases increased the lateral plastic resistance (in comparison with pinned bases) by approximately one hundred percent. Use of heavy roofing, compared with lightweight roofing, had little effect on the lateral resistance of the truss frame design but increased the continuous welded frame lateral resistance by about 100 percent. The use of a continuous welded frame

instead of a truss frame increased the lateral resistance by a factor of between 3 and 7 for the various other conditions that were considered. One reason for the superiority of continuous welded frame construction as compared with truss frame construction was the fact that the lack of crane loads resulted in columns that were quite small in the case of the truss designs. In the case of continuous welded frame design, bending moments are induced in the columns by the roof loads and this results in much heavier column selections with corresponding increase in lateral resistance.

In later special studies of modifications that might be made in conventional peaked roof truss frame design for a warehouse building with medium crane loadings, it was found that the use of a single column with crane brackets gave appreciable increase in strength as compared with a stepped column wherein the crane runway beams were supported by their individual columns. The use of crane runway brackets increases the design bending moment in the columns and this results in heavier design and corresponding increase in lateral strength.

The John A. Roeblings Sons Company has made a load-deflection test to failure on 1-1/4" inch galvanized wire strands that were analyzed for their effective increase in lateral strength. These were assumed as attached to the top of the exterior columns and were anchored in the ground outside the building. In the case of the truss frame design with light roofing and pinned column bases the wire strand multiplied resistance by a factor of between 6 and 7. In the case of the much stronger continuous welded frame design with heavy roof and fixed column bases the addition of wire strand improved the lateral plastic resistance by about 50 per cent.

As an alternate to the use of strands, steel rods could be used, in which case the threaded ends should be upset so as to insure yielding throughout the length of the rod and thus provide maximum plastic deflection of the structure. Comparing the extreme range of these designs, including the use of special strand reinforcement, there was found to be a ratio of 25 to 1 between the strongest and weakest. If such a change in lateral resistance could be generally effected, it would probably reduce the <u>area</u> of total destruction under any nuclear blast by a factor of between 8 and 10.

In summary, various levels of main frame resistance are shown pictorially in Fig. 3a, 3b, and 3c under the headings poor resistance, improved resistance, and best open frame resistance, respectively. As a logical extension of the ideas previously discussed with regard to single-bay buildings, Fig. 4 shows adaptation to a three-bay structure in which it is assumed that through clearance is not necessary in one of the bays. In this case diagonal bracing, as shown, would greatly increase the lateral strength of the structure. As an alternate method of increasing lateral strength by use of cross bracing and at the same time maintaining open bay area ways, Fig. 5 suggests the use of a flat concrete roof acting as a beam in horizontal plan to transfer the loads into end frames where bracing would transfer the roof reactions to the foundation. This, of course, is an obvious and often used procedure in building construction as well as in bridge construction.

To check by dynamic analysis some of the suggestions that have been made, the three different steel frames that were illustrated in Fig. 3 as examples of poor, improved, and best open frame resistance were studied for various intensities of loading resulting from a nominal 20 KT atomic bomb burst at 2000 foot altitude above ground zero. The dimensions given to these frames permitted 60 foot clear space horizontally and 20 foot clear space vertically so that they all met the same space requirements. The siding and

girts were the same for all three frames. Corrugated asbestos was used for siding. All frames were designed for vertical snow load of 40 psf. Analyses were based on unclassified information that is generally available,[1,5] and were limited to a typical interior bent of a single bay frame.

The following is a general description of the three frames:

Frame A (Fig. 3a) was a conventional peaked roof structure based on a design in the AISC Structural Drafting Textbook, Vol. II. Estimates of mass and projected areas were obtained from this design. Frame A was assumed to have a corrugated asbestos roof and the column bases were assumed as pinned.

Frame B, (Fig. 3b) was designed for concrete roof slab and with column bases fixed. However, it is a conventional truss frame design and, incidentally, was identical to a frame designed for and considered in an earlier investigation.[2] Estimates of mass and projected areas of bracing including eave trusses were taken from a similar design presented in the AISC Structural Drafting Textbook. Volume II.

Frame C, a continuous welded structure, was designed for a 20 psf wind load. Although not originally contemplated, it turned out that this frame very nearly met the proposed plastic design strength corresponding to a constant drag load of 1400 psf lateral load on the vertically projected skeleton area. The roof was a five inch concrete slab, cast in place, with the upper flanges of the purlins imbedded in the concrete to provide for their continuous lateral support. In this frame, alone of the three, width-thickness ratios of sections were selected in accordance with the recommendations made in this report, thus assuring maximum plastic resistance. Column bases were assumed to be fixed.

Using information available in unclassified government publications[1,5] the equivalent drag load at the top of the columns in each frame was determined as a function of time. Then, using the numerical procedure proposed by Newmark,[12] an approximate dynamic analysis was made of each frame to determine the maximum permanent set that it would experience at different distances from ground zero of a "nominal" atomic bomb burst. In order to make the dynamic analysis it was, of course, necessary to determine a "resistance" curve. The resistance curve for these frames was a plot of equivalent load vs. deflection at the top of the column just at the roof eaves. As recommended initially in this report the "dynamic yield-point" for structural steel was taken as 42 ksi thus allowing for an increase of 9 ksi above the specification yield-point of 33 ksi.

The effect of dead load in reducing the lateral resistance of frames in the plastic range was neglected in these studies since it would be negligible in the region where total permanent deflection was small. In other studies wherein large deflections were considered the effect of dead load has been found to be a considerable one.

The principal features of the design and dynamic analysis of Frame C at 3000 feet from a 2000 foot altitude burst of a nominal atomic bomb are given in an appendix to this report as an illustrative example.

This analysis shows that Frame C had a permanent set of between two and three inches at 3000 feet and remained elastic at 4000 feet from ground zero thus demonstrating the fact that frames designed for the proposed lateral drag load of 1400 lbs. per square foot of vertically projected surface would survive with little damage at distances even less than 3000 feet from ground zero of a nominal atomic bomb burst. Frame B, on the other hand, showed more than an 88 inch permanent deflection at 3000 feet and at 6000 feet had a

permanent deflection of two inches. Frame A, embodying poor features of design insofar as resistance to atomic blast is concerned, had more than an 88 inch permanent set at 6000 feet from ground zero.

The foregoing are, of course, calculated estimates and are not based on actual tests.

Personnel Shelters

Although design studies for personnel shelters are outside the scope of this report such reinforced concrete shelters should be provided adjacent to or within structures that are designed with frangible walls as proposed herein. Recommendations for shelter design can be obtained from the Federal Civil Defense Administration Headquarters in Washington, D. C., or through local Civil Defense offices.

The best defense against atomic blast attack is to house industrial plants underground in artificial or natural caverns. This approach is, in fact, extensively used in some countries such as Sweden. As mentioned herein, in the case of many facilities of key importance it may be desirable to design buildings to completely withstand atomic blast at a given desired critical distance. Another facet in the overall approach is the desirability of wide dispersion of industrial targets. Such dispersion has been encouraged by the Federal Civil Defense Administration. However, with or without dispersion, the suggestions for one-story industrial buildings[6] that have been made herein will decrease the total damage in any attack.

CONCLUSIONS

This paper has summarized specific recommendations for the design of one-story industrial building frames wherein the building contents are of a type that may survive intense wind gust accompanying atomic blast without serious damage. Recommendations are given for the design of wall, roof, and mainframe construction. Corroboration of the recommendations is made by means of specific design examples and the dynamic response of these designs to blast loads is summarized. An illustrative analysis and design for one of these cases is presented in the appendix.

It is found that the structural design of drag-type structures can be reduced for all practical purposes to design for an equivalent static drag load if the object is survival of the structure with small permanent deflections. The achievement of overall balanced plastic strength, with no weak links in the chain of force transfer, is much more important than precise dynamic analyses if the primary goal is reduction of wartime damage.

ACKNOWLEDGMENTS

This study has been supported primarily by a grant from the Rackham School of Graduate Studies, University of Michigan and was a research project under the Civil Engineering Department of that University. In addition to studies made under the Rackham grant the paper includes recommendations that resulted from other similar projects on industrial building frames

6. Similar studies suggesting the use of frangible walls in tier buildings, utilizing a cylindrical central core to provide both strength and personnel shelter, have been suggested by one of the authors.(13)

made for Lehigh University and Massachusetts Institute of Technology. Acknowledgment is made to the many research assistants who have been engaged on one or more of these projects, including R. C. Byce, L. S. Hu, C. Muktabhant, H. H. Tung, C. Salmon, L. Schenker, and, in special connection with the preparation of this paper, Mr. Nelson Isada.

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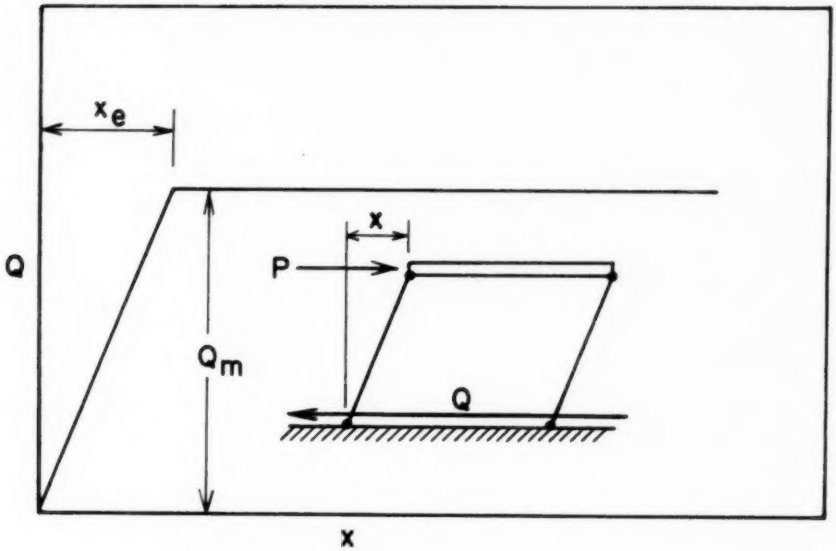
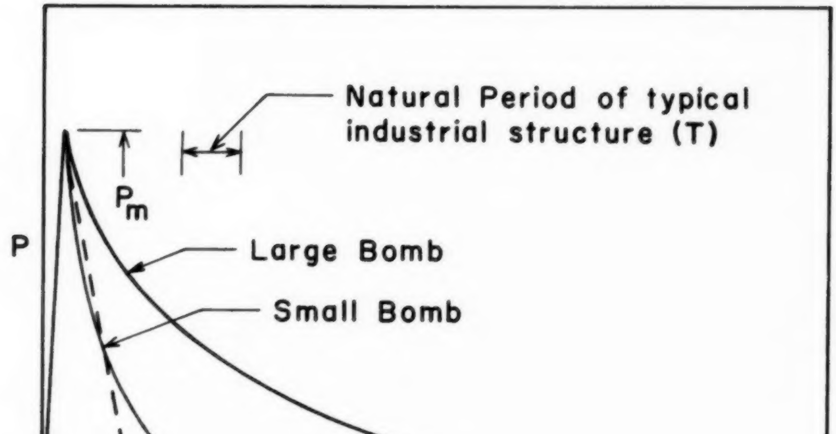


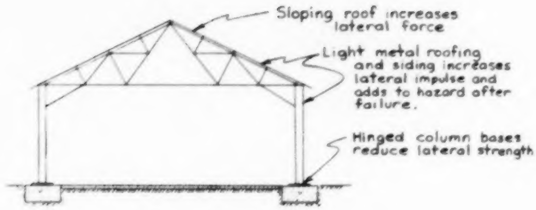
x_e
Q
Q_m
x
P
Q
x

Fig. 1



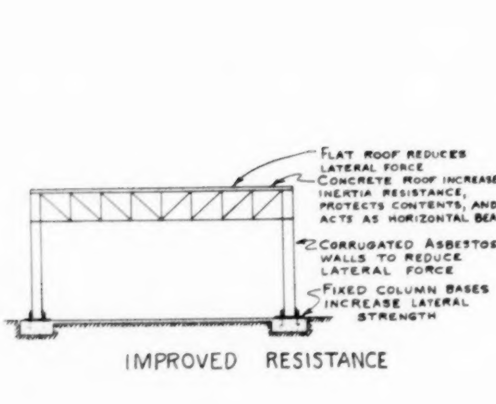
Natural Period of typical industrial structure (T)
P_m
P
Large Bomb
Small Bomb
t_1
t

Fig. 2



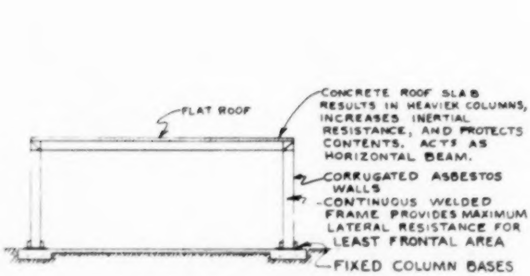
POOR RESISTANCE

Fig. 3a



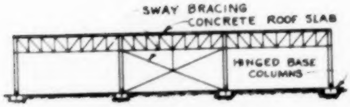
IMPROVED RESISTANCE

Fig. 3b



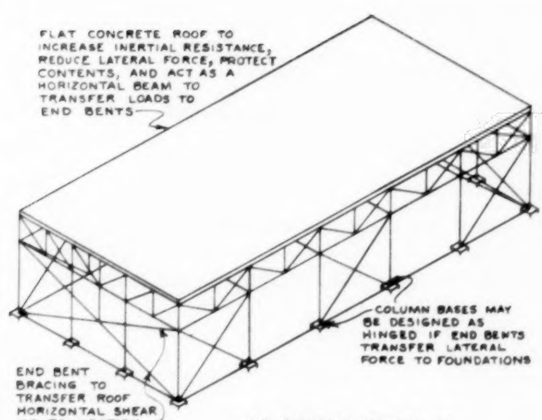
BEST OPEN FRAME RESISTANCE

Fig. 3c



MULTI-BAY BUILDINGS

Fig. 4



USE OF END BENTS TO RESIST LATERAL FORCE

Fig. 5

APPENDIX

In the following example a rigid frame structure located 3000 feet from ground zero is analyzed for response to exposure to a "nominal" atomic bomb exploded at 2000 feet altitude.

The corrugated asbestos siding fails rapidly, transferring a relatively small impulse to the frame in comparison with impulse due to drag. Therefore, the load on the frame will be considered to be entirely of the drag type.

Design of Typical Bay

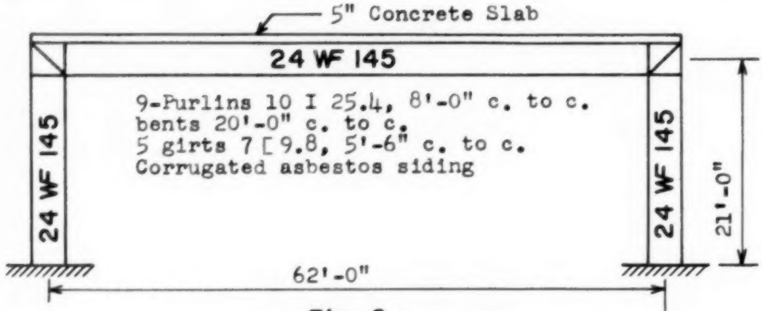


Fig. 6

Effective Mass

Estimated weight of one bay

$W_{(roof)}$ (slab, roofing, beam, purlins) = 102 kips

$W_{(walls)}$ (girts, columns) = 9 kips

$$M_{(effective)} = \frac{1}{g}\left[W_{(roof)} + \frac{1}{3} W_{(walls)}\right]$$

$$M_{(eff.)} = \frac{1}{386}\left[102 + \frac{9}{3}\right] = 0.272 \text{ kips sec}^2 \text{ in}^{-1}$$

Resistance Curve

Compute resistance of frame to a concentrated lateral force applied at top of column.

Using moment distribution, the end moments after distribution for a 1" deflection are:

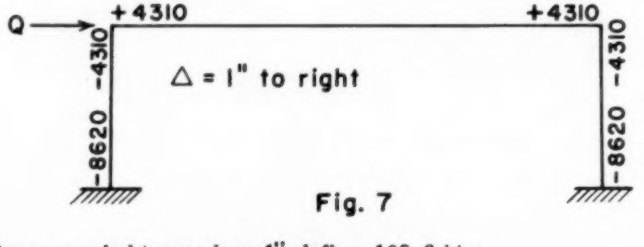


Fig. 7

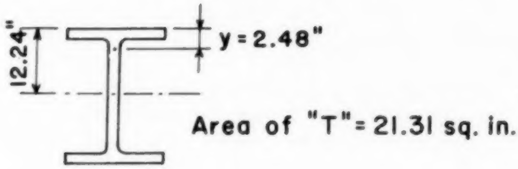
Force needed to produce 1" defl. = 102-8 kips
Elastic resistance = 102.8 kips/in

Plastic Hinge Moment for 24 WF 145:

Assume dynamic yield value of 42 ksi. (based on static yield of 33 ksi. and time to reach yield 0.1 second).

$M_p = f_y Z$, Z = plastic section modulus

Z is calculated from properties of ST 24 WF 145



$$Z = 2 \; [21.31 \; (12.24-2.48)] = 416 \text{ in}^3$$

$$M_p = 42 \times 416 = 17{,}500 \text{ in-kips}$$

$$Q_{max} = 4 \frac{M_p}{h} = \frac{4(17{,}500)}{21 \times 12} = 278 \text{ kips}$$

The foregoing leads to a simplified resistance curve, suitable for approximate analyses, but neglecting the intermediate hinge formation calculations. This approximate resistance curve is indicated on Fig. 8 by the dashed line wherever it differs from a more exact solution (details not given) which is shown by a solid line throughout.

Load Curve

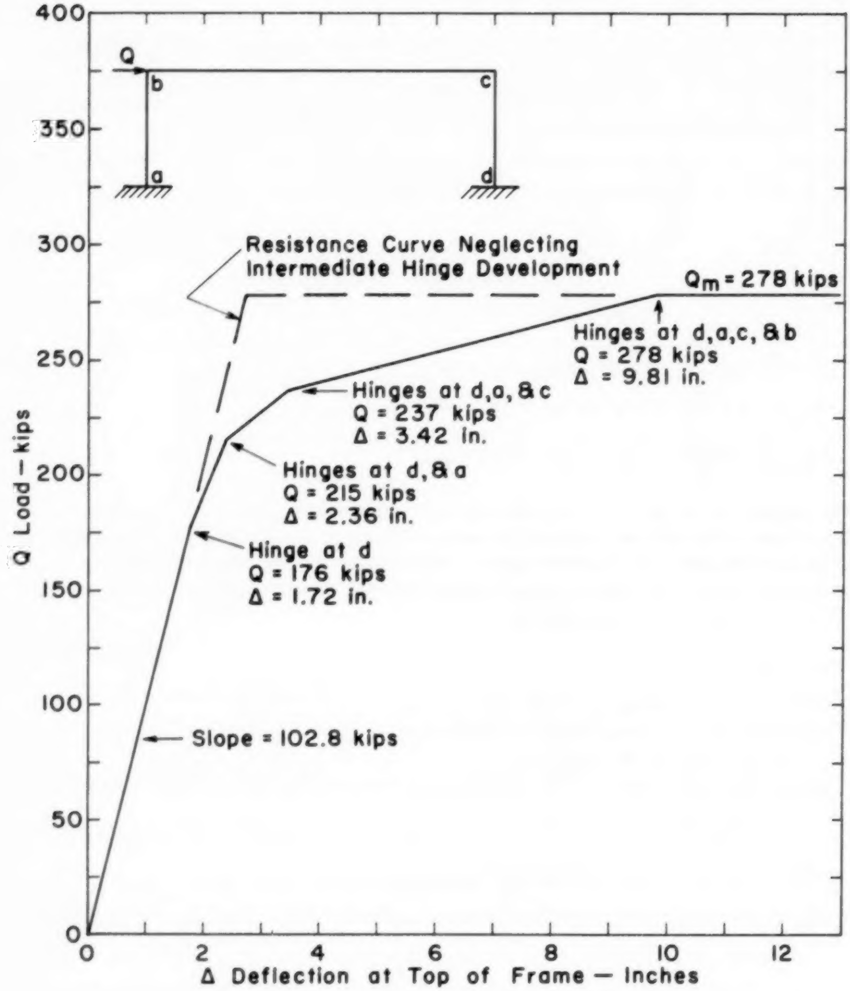
Peak Overpressure at 3000 GZ = 16 psi.
Duration of Positive Phase = 0.62 sec.
from Table 5.45, Ref. #1.
Equation 5.19.1 Ref. #1 is used to plot Overpressure vs. Time Curve.

$$p_s = p_s^o \; (1 - t/t_o) \; e^{-t/t_o}$$

Where p_s is the overpressure at time t,

p_s^o is the peak overpressure (side-on), and t_o is the duration of positive phase.

t/t_o	p_s/p_s^o	t (sec)	p_s (psi)
0	1	0	16
0.05	0.904	0.03	14.5
0.1	0.815	0.06	13.0
0.2	0.655	0.12	10.5
0.3	0.519	0.19	8.3
0.4	0.402	0.25	6.4
0.5	0.304	0.31	4.9
0.6	0.220	0.37	3.5
0.7	0.149	0.43	2.4
0.8	0.090	0.50	1.4
0.9	0.041	0.56	0.7
1.0	0	0.62	0



RESISTANCE CURVE

Fig. 8

Equation 7 page 63, Ref. #5 is used to plot Drag Pressure vs. Time Curve.

$$P_d = \frac{5}{2} \; C_d \left(\frac{p_s^{\,2}}{102.9 + p_s} \right)$$

Where P_d is the drag pressure due to wind velocity behind the shock front, C_d is the drag coefficient, assumed to be 2 in this example, and p_s is the overpressure at time t.

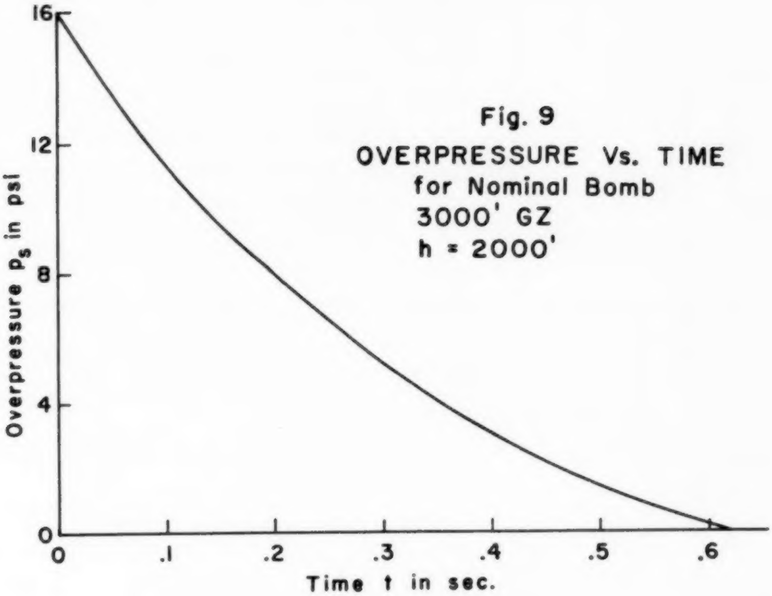


Fig. 9
OVERPRESSURE Vs. TIME
for Nominal Bomb
3000' GZ
h = 2000'

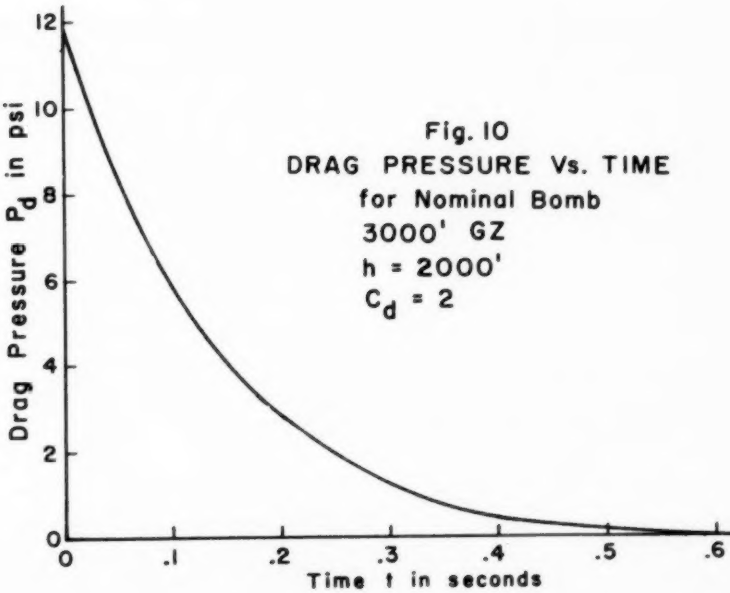


Fig. 10
DRAG PRESSURE Vs. TIME
for Nominal Bomb
3000' GZ
h = 2000'
C_d = 2

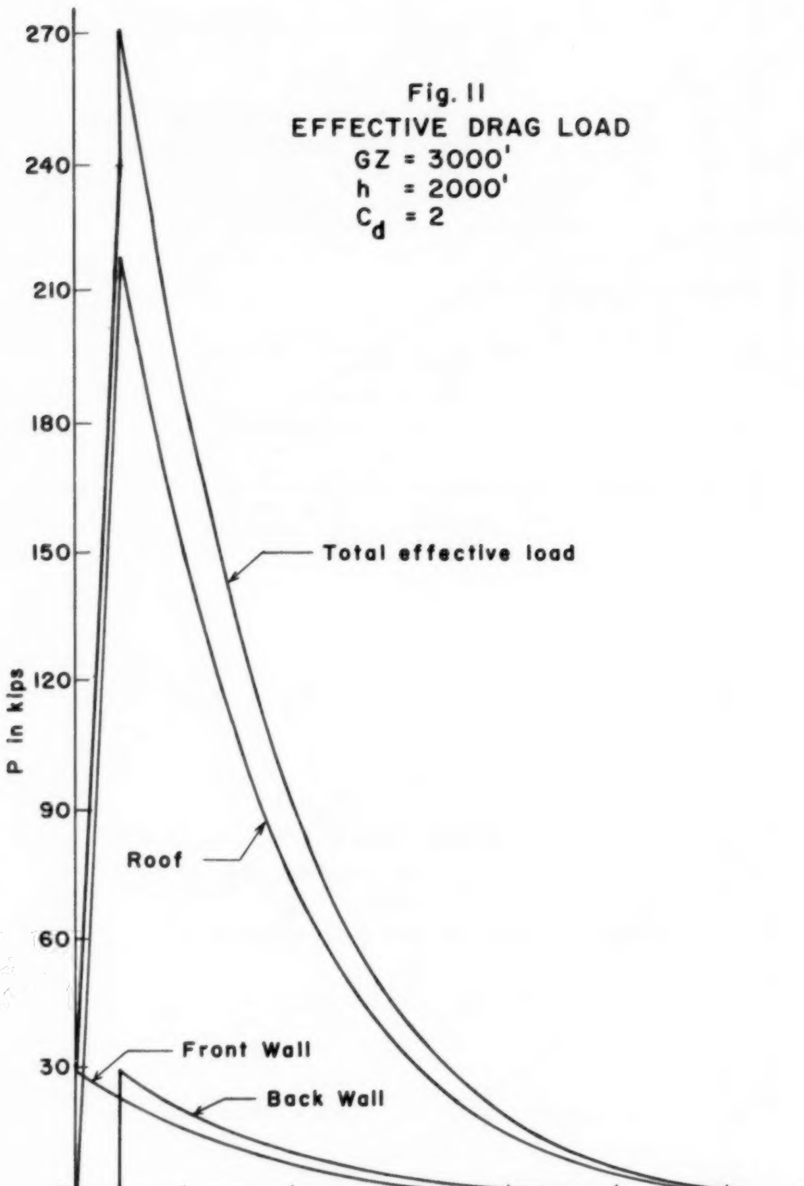


Fig. 11
EFFECTIVE DRAG LOAD
GZ = 3000'
h = 2000'
C_d = 2
Total effective load
Roof
Front Wall
Back Wall
P in kips
270
240
210
180
150
120
90
60
30
0
0
.04
.1
.2
.3
.4
.5
.6
Time t in seconds

Estimated vertical projected area for one bay:

at roof level, (purlins, slab) = $A_{(roof)} = 158\ ft^2$

one wall, (girts, column) = $A_{(wall)} = 38\ ft^2$

Effective load on wall = $1/2 \times A_{(wall)} \times P_d$

Average effective load on roof = $A_{(roof)} \times P_{d\ (avg.)}$

Shock Velocity = 1700 ft/sec - Figure 5.22 Ref #1

Time for Shock to cross building = L/V

$$\frac{L}{V} = \frac{64}{1700} = 0.04 \text{ Sec.}$$

Numerical Analysis of Structure (See Reference 12 for details of procedure).

$M_{(eff.)} = 0.272$ Kip $Sec^2\ In^{-1}$

$$\text{Natural period} = T = 2\pi \frac{M}{K} = 2\pi \sqrt{\frac{0.272}{102.8}} = 0.32 \text{ sec.}$$

$$\tau = \frac{T}{8} = \frac{0.32}{8} = 0.04 \text{ sec.}$$

P_{eff} and Q_{eff} shown as curves

$$a = \frac{P-Q}{M} = 3.68\ (P-Q)$$

$$v'' = v' + \frac{\tau}{2}\ (a' + a'')$$

$$x'' = x' + v\ \tau + \frac{a'\tau^2}{3} + \frac{a''\tau^2}{6}$$

Based on approximate resistance curve

t(sec)	τ(sec)	P(Kips)	Trial Q(Kips)	P-Q	a(in/sec^2)	v(in/sec)	x(in)	Q
0	0	29	0	29	107	0	0	0
0.04	0.04	271	30	241	887		0.3	31
			31	240	884	20	0.3	31
0.08	0.04	207	107	100	368		1.7	175
			170	37	136		1.6	165
			165	32	118	40	1.6	165
0.12	0.04	162	262	-100	-368		3.2	278
			278	-116	-427	34	3.1	278
0.16	0.04	123	278	-155	-570	14	4.1	278
0.20	0.04	91	278	-187	-687	-11	4.2	278
0.18	0.02	105	278	-173	-636	2	4.3	278

Max. defl. = 4.3"
Elastic defl. = 2.5"
Permanent Set = 1.8"

(Based on more accurate resistance curve)
Same as above up to t = 0.08 sec.

0.12	0.04	162	262	-100	-368		3.2	232
			232	- 70	-258	37	3.2	232
0.16	0.04	123	243	-120	-432	23	4.4	243
0.20	0.04	91	251	-160	-589		4.9	247
			247	-156	-574	3	5.0	247
0.22	0.02	84	249	-165	-607		4.9	247
			247	-163	-600	-12	4.9	247

Max. defl. = 5.0"
Elastic defl. = 1.7"
Permanent set = 2.5"

Notation

Symbol	Definition	Dimension
a'	Acceleration at beginning of interval in Numerical Analysis.	in./sec.^{2}
a''	Acceleration at end of Interval in Numerical Analysis.	in./sec.^{2}
C_d	Drag Coefficient.	non-dimensional
g	Acceleration of Gravity.	in./sec.^{2}
h	Height of Burst.	ft.
L	Length of Structure in direction of Shock Motion.	ft.
$M_{(eff.)}$	Effective Mass of Structure.	Kip Sec^2/in.
M_p	Plastic Hinge Moment.	in.-Kips
P_d	Drag Pressure due to wind velocity behind shock front.	psi
p_s	Overpressure behind shock front at any given time.	psi
p_s^o	Peak Overpressure at given Distance from Ground Zero.	psi
Q	Equivalent Resistance.	Kips
T	Natural period of Structure	Sec.
t_o	Duration of Positive Phase.	Sec.
V	Velocity of Shock Front	ft./sec.
v'	Velocity at beginning of interval in Numerical Analysis.	ft./sec.
v''	Velocity at end of interval in Numerical Analysis.	ft./sec.
x'	Deflection at beginning of interval in Numerical Analysis.	in.
x''	Deflection at end of interval in Numerical Analysis.	in.
Z	Plastic Section Modulus.	in.^{3}
Δ	Lateral Deflection at top of Column.	in.
τ	Interval in Numerical Analysis.	Sec.